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Vol. V NOVEMBER, 1918 No. 9

BOSTON SOCIETY OF CIVIL ENGINEERS

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PROCEEDINGS

PAPER IN THIS NUMBER.

"Report of Committee on Run-off."

CURRENT DISCUSSIONS.

Paper.	Author.	Published.	Discussion Closes.
"Notes on the Construction of the Mystic River Bridge of the Boston Elevated Railway."	Clarence T. Fernald.	Oct.	Dec. 10.
"Mystic River Bridge — Description of Superstructure."	John C. Moses.	Oct.	Dec. 10.

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Contributors are hereby notified that proof will not be submitted to them for examination unless requested before the 10th of the month preceding the month of publication.

APPLICATIONS FOR MEMBERSHIP.

[November 15, 1918.]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

Calderara, Charles Anthony, Rockland, Mass. (Age 28, b. Quincy, Mass.) Graduate of Dartmouth College, 1913, degree of B.S.; graduate of Mass. Inst. of Technology, 1915, sanitary engineering course. From June, 1915, to May, 1916, laborer, engineer's assistant, quantity man, gang boss and timekeeper with Aberthaw Construction Co.; from May, 1916, to April, 1917, sanitary inspector with National Canners' Association, Sardine Section; from April, 1917, to April, 1918, field and office assistant with Stewart & McCurdy, Civil Engrs., Akron, Ohio; from May, 1918, to date, draftsman and assistant in progress department with Fay, Spofford & Thorndike, Boston Army Supply Base. Refers to G. D. Emerson, Dwight Porter, H. F. Sawtelle, C. M. Spofford and P. B. Walker.

Christie, Madison Walter, Somerville, Mass. (Age 30, b. Charlestown, Mass.) Graduate of Mass. Inst. of Technology, 1913, civil engineering course. After graduation employed by New England Concrete Construction Co. on reinforced building construction; in March, 1914, employed by B. & M. R. R. Co. on railroad survey; from August, 1914, to November, 1915, with same company, office of engineer of structures; in November, 1915, employed by B. & A. R. R. Co., office of engineer of structures; from November, 1916, to August, 1917, with U. S. Navy Dept., Washington, D. D., on industrial plants; from September, 1917, to February, 1918, with Monks & Johnson on Squantum Plant; from February, 1918, to date, with Fay, Spofford & Thorndike on Boston Army Supply Base. Refers to C. R. Berry, H. A. Gray, H. F. Sawtelle, C. M. Spofford and P. B. Walker.

Freeman, Edward Alden, Newton Centre, Mass. (Age 22, b. Concord, N. H.) Graduate of Harvard College, 1917, having specialized in engineering sciences; graduate of Mass. Inst. of Technology, October, 1918, civil engineering course. From October 8, 1918, to date, with Fay, Spofford & Thorndike in field office (progress dept.), Boston Army Supply Base. Refers to H. J. Hughes, L. J. Johnson, Dwight Porter, C. M. Spofford and P. B. Walker.

ROLL OF HONOR.*

* Members who are in the service and have not yet reported that fact to the Secretary are earnestly requested to do so, stating branch of service, rank and military address.

ADDITIONS.

GREGORY, JOHN H. Captain, Sanitary Corps, U. S. A., 53 Christopher St., Montclair, N. J.

HYDE, EDWARD R. Captain, Engr. Corps, U. S. A., Camp Humphreys, Va.

NEWMAN, ROLF R. Captain, 403d Engrs., U. S. A., Fort Douglas, Utah.

ROBINSON, BYRON A. Captain, Q. M. C., U. S. A., Raritan Arsenal, 63 Carsen Ave., Metuchen, N. J.

REVISIONS.

COBURN, WILLIAM H. Captain, Gas Defense Div., Chemical Warfare Service, care B. F. Goodrich Co., Akron, Ohio.

KENDALL, THEODORE R. 1st Lieutenant, Sanitary Corps, U. S. A., Watervliet Arsenal, Watervliet, N. Y.

ROBINSON, ASHLEY Q. Musician, 334th Field Artillery Band, Am. Ex. Force, via New York, N. Y.

WARING, CHARLES T. Major, Aircraft Acceptance Park No. 2, 35 Eaton Place, London, S. W. 1, England.

WESTON, ARTHUR D. Captain, 34th Engrs., in charge of all water-works materials, Am. Ex. Force, via New York, N. Y.

LIST OF MEMBERS.

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CHANGES OF ADDRESS.

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Care Monks & Johnson, cor. 19th and Wylie Sts., Philadelphia, Pa.
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THORPE, GEORGE H.......................7 Nelson St., Lawrence, Mass.
WALKER, FRANK B.
Care Fay, Spofford & Thorndike, Army Supply Base, Boston, Mass.
YOUNG, ERVING M., Picric Acid Plant, care Major Blanchard, Brunswick, Ga.

DEATH.

DODD, CHARLES H...................................October 27, 1918.

EMPLOYMENT BUREAU.

The Board of Government has established an employment bureau for the Society, to be a medium for securing positions for its members and applicants for membership, and also for furnishing employees to members and others desiring men capable of filling responsible positions.

At the Society room two lists are kept on file, one of *positions available* and the other of *men available*, giving in each case detailed information in relation thereto.

Positions Available.

Transitman and assistant wanted in town engineer's office. Apply to J. Henry Duffy, Town Engineer, Lexington, Mass.

Draftsman wanted with some knowledge of building and construction work and capacity to develop along plant engineering lines. Apply to J. Hanson Mitchell, Plant Engineer, Hamilton Woolen Co., Southbridge, Mass.

LIBRARY NOTES.

Recent Additions to the Library.

U. S. Government Reports.

Abstracts of Current Decisions on Mines and Mining, September to December, 1917. J. W. Thompson.

Atlantic Coast Tide Tables for Eastern North America for 1918.

Fossil Flora from Frontier Formation of Southwestern Wyoming. F. H. Knowlton.

Highway Cost Keeping. James J. Tobin and A. R. Losh.

Measuring the Temperature of Gases in Boiler Settings. Henry Kreisinger and J. F. Barkley.

Quarry Accidents in United States during Calendar Year 1916. Albert H. Fay.

Recovery of Gasoline from Natural Gas by Compression and Refrigeration. W. P. Dykema.

Slag Viscosity Tables for Blast-Furnace Work. Alexander L. Feild and P. H. Royster.

State Reports.

Ohio. Technical Reports of Miami Conservancy District, Part II: History of Miami Flood Control Project. C. A. Bock.

Municipal Reports.

New York, N. Y. Record in Suit of People of State of New York, Complainants, *v.* State of New Jersey and Passaic Valley Sewerage Commissioners, Defendants. 5 vols.

Northampton, Mass. Annual Reports of City Officers for 1917.

Miscellaneous.

Canada, Department of Mines: Report on Building and Ornamental Stones of Canada, Vol. V, British Columbia, by Wm. A. Parks; Mineral Springs of Canada, Part II, Chemical Character of Some Canadian Mineral Springs, by R. T. Elworthy.

E. I. du Pont de Nemours & Co.: Blasters' Handbook.

National Board of Fire Underwriters: Reports and Supplements on following cities: Highland Park, N. J., New Brunswick, N. J., Philadelphia, Pa., Pittsburgh, Pa., Richmond, Va., Scranton, Pa., Waterbury, Conn.

LIBRARY COMMITTEE.

Vol. V NOVEMBER, 1918 No. 9

BOSTON SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications

PROGRESS REPORT OF COMMITTEE ON RUN-OFF.*

THE Boston Society of Civil Engineers' Committee on Run-off, appointed December 20, 1916, made a report to the Society which was published in the JOURNAL for April, 1918. Since that time the committee has met regularly each month with constantly decreased attendance, due to the fact that each of the members of the committee is doing two men's work in order to keep going the work of his office. It appeared almost necessary to give up active meetings of the committee until the end of the war, and some method was sought to keep up the interest in the work without requiring attendance on the meetings.

At a meeting of the Run-off Committee held September 18, 1918, the commitee voted to ask the Board of Directors of the Boston Society of Civil Engineers to publish advance chapters of its reports in bulletins, with the idea of making progress on the reports and giving members of the Society and others an opportunity to criticize and amplify the material of the subjects discussed.

The Secretary of the Society very kindly suggested that some of the advance material which had been prepared and discussed by the committee might be presented for discussion by members and others interested, in the form of advance copies published in the JOURNAL. The chairman has therefore prepared this statement, and offers a number of papers on different subjects which he hopes even in war times will call out some discussions which will be of advantage to the members of the committee when they have an opportunity to get all these matters in shape for a report. The list is as follows:

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* Members of committee: Arthur T. Safford, chairman; Charles H. Pierce, secretary; Harold S. Boardman, vice chairman; A. C. Eaton, X. Henry Goodnough, Richard A. Hale, Charles W. Sherman, Herbe A. Moody, W. Frank Uhl, Joseph F. Wilber, Dana M. Wood.

Glossary of terms.
Use of the current meter in stream gaging.
0.2 and 0.8 method in power canals.
Precipitation, evaporation and run-off.
Effects of ice on river discharge.
Methods to be used in compilation of data.

GLOSSARY OF TERMS.

1. *Precipitation* is water that passes out of the atmosphere, in liquid or solid state, to the surface of the earth or bodies of surface water. It includes rain, snow, hail, sleet, dew and frost. It is the process by which atmospheric moisture becomes surface water. Rainfall and snowfall make up precipitation as ordinarily used.

(*a*) *Rainfall* is sometimes used in the sense of "precipitation," but it should become more general practice to limit its use to atmospheric moisture precipitated in the form of rain. Its common use in the broader sense undoubtedly is the result of the practice of measuring all forms of atmospheric precipitation in terms of "inches of rainfall."

(*b*) *Snowfall* is considered as snow, hail, sleet and other forms of precipitation, except rain, measured in depth in inches as it falls, but usually appears in records as its water equivalent.

(*c*) *Snow residuum* is the accumulated snow on the ground surface at the end of any period, or its water equivalent.

(*d*) *Water equivalent* is the depth of water in inches to which the snow on a given area would melt.

(*e*) *An isohyetal map* shows variation in amount of average precipitation over contiguous areas and *isohyetal lines* are used to connect contiguous points of equal precipitation, similar to contour lines.

2. *Evaporation* is the return of water from the earth or water

surface to the atmosphere. It takes place in three or more ways, as from —

(*a*) Water, ice and snow surfaces; (*b*) land surfaces; and (*c*) vegetation, in the form of transpiration. Both evaporation and precipitation are the results of differences in temperature and atmospheric pressure.

3. *Seepage* or *percolation* of water is the movement of water underground, produced by hydrostatic pressure.

4. *Absorption* comprises all processes by which water is taken up by the earth's surface.

5. *Leakage* is the movement of water through artificial structures, and is produced by hydrostatic pressure.

6. *Discharge* is a general term for the rate of flow of water. In water-power practice, it is usually expressed in cubic feet per second (frequently abbreviated to second-feet); in water-works practice, it is customarily expressed in terms of million gallons per twenty-four hours.

7. The *drainage area* of a stream at a given point is the area of the surface of the earth which contributes its run-off to the stream above that point. Synonyms: Catchment area, watershed. The area used may be either gross, or the net exclusive of lake and pond areas, but it is assumed to be the former unless specified to the contrary.

8. The *run-off* of a given area of the surface of the earth is the quantity of water that is discharged from that area. Ordinary units of run-off are second-feet per square mile or run-off depth in inches.

(*a*) *Cubic foot per second* or *second-foot* is a unit of discharge of water flowing in a channel of one square foot in cross section at an average velocity of one foot per second.

(*b*) *Second-foot per square mile* is a unit of discharge indicating the average number of cubic feet of water flowing per second from each square mile of area drained, on the assumption that the run-off is distributed uniformly both as regards time and area.

(*c*) *Run-off depth in inches* is a unit of run-off indicating the depth to which the drainage area would be covered if all of the water flowing from it in a given period of time were conserved and uniformly distributed on the surface. It is used for comparing run-off with precipitation, which is usually expressed in depth in inches.

9. The *yield* of a drainage area at any definite point is the amount of surface and underground waters contributed by it to a drainage system. If any of these waters are retained, either by natural or artificial storage within the watershed, the yield will not be the same as the run-off during the same period of time.

10. *Storage.*—The word usually means—

(*a*) *Artificial storage*, which is that water capacity which may be available to increase the extremely low flows for several days, weeks, months or even years. It is the retention of the flood waters for use during times of scarcity, and is usually located near the headwaters of a stream.

(*b*) There is usually some *natural storage* in the form of swamps, lakes and ponds, but its effect on stream flow cannot easily be separated from that of the rest of the drainage area.

11. *Pondage* is that water capacity created by any dam which tends to take care of the variations in draft at a water-power development. It is the retention for one or more days of the flow of a stream during hours of light load, for use during hours of heavy load.

12. *Factors:*

(*a*) *Load factor* is the ratio of the average to the maximum power load on a water-power plant during a given period of time, as a day, month or year. To be specific, this period should always be stated, as "daily load factor," etc. Frequently when plants operate for a period less than twenty-four hours, the load factor is expressed in terms of "load factor for hours when running"; but for purposes of stream-flow analysis all such factors should be reduced to the twenty-four hour basis.

In determining load factor, the "maximum load" should be the heaviest observed during the period. Usually, daily log records are kept, and switchboard instruments, when available, are read every half hour, or hour, and the highest of these observations is taken as the maximum.

The "average load" is the total energy output divided by the proper time unit.

(*b*) *Capacity factor* is the ratio of the average load in a given time period, as a day, to the total installed rated capacity of the equipment.

(*c*) *Utilization factor* is that coefficient by which the approximate theoretical energy-outputmust be multiplied to correspond to the actual delivered energy. It accounts for losses of all kinds, and for variations in operating conditions other than those included in the average plant efficiency.

The approximate energy available in a stream can be estimated by the following formulas.

I. Net horse-power at the water-wheel realizing 80 per cent. of the theoretical power $= \frac{\text{cubic feet per second x head}}{11}$.

If other than 80 per cent. is used as the efficiency of the equipment, then the formula should be —

$$\text{Net horse-power} = \frac{\text{cubic feet per second x head x efficiency}}{8.8}.$$

II. Electrical horse-power (73.3 over-all efficiency) =

$$\frac{\text{cubic feet per second x head}}{12};$$

or,

$$\text{III. Kilowatts} = \frac{\text{cubic feet per second x head}}{16}.$$

Losses allowed for in the utilization factor are leakage and waste in operation, etc.

Changes in efficiencies may be due to variations in load or variations in head from the average assumed for use in the formulas.

13. *River gaging stations.*

(*a*) *Control* is the natural or artificial barrier, reef of ledges or other channel conditions below a gaging station which primarily affects and controls the relation between gage height and discharge at the station. It should be noted that the control may not be the same section for all stages.

(*b*) *Rating table* for a gaging station is a table showing the discharge corresponding to various gage heights at the station.

(*c*) *Rating curve* for a gaging station is a curve of relations between gage height and discharge.

(*d*) *Stage-discharge relation* is the relation between gage height and discharge as shown by the rating curve, q. v.

(*e*) *Water-stage recorder* is an instrument for automatically recording the elevation of the water surface, and the corresponding time.

14. *Regulation* is the artificial manipulation of the run-off of a stream.

15. *Fall* is the vertical distance between water surface levels at two points on a stream under static flow conditions.

(*a*) *Gross head* is that portion of the fall which is available for producing power.

(*b*) *Net* or *effective head* is the residue left to produce energy after deducting all losses of head. These losses may be due to friction, irregularities of section and other causes.

(*c*) *Slope* is the total fall between two points divided by their distance apart, expressed in the same units.

16. *Dams:*

(*a*) *Crest of a dam* is, in general, the top of the fixed overflow structure. In some cases there may be overflows at several elevations.

(*b*) *Height of a dam* should be specifically defined as either: I, from the bottom of foundations to the top of the structure; or II, from the original bed of the river to the crest of the dam.

17. *Backwater.* — This is the general term applied to the raising of water upstream from an obstruction to higher elevations than it would stand under similar conditions of flow with no obstruction. Such an obstruction may be a permanent structure like a dam or bridge pier, or may be a temporary obstruction like ice or logs. An unusually heavy discharge from a tributary may cause backwater in the main stream or vice versa.

(*a*) *Backwater curve* is a profile of the water surface upstream from an obstruction between such obstruction and the point where the backwater ceases. Such a curve or profile is used in determining the elevation of the water at any point under known or assumed conditions of flow. Each different set of conditions produces a different backwater curve.

(*b*) *Backwater effect* is the increase in water-surface elevation caused by an obstruction, and varies with the discharge. It may be obtained at any point by a comparison of the backwater curve and the natural profile of the stream with no obstruction, each referred to the same discharge. In discussions on this subject the term *backwater effect* is sometimes shortened to *backwater*.

THE USE OF THE CURRENT METER IN STREAM GAGING.*

About thirty years ago, at the time of starting the work of the Irrigation Survey, which later developed into the Reclamation Service, attention was called to the necessity of devising some simple means of accurately measuring the flow of water in natural streams. The first annual report of the Irrigation Survey, J. W. Powell, director, contains a very complete history of the early stages of hydrometric work, including a description of the training school for hydrographers which was held at Embudo, N. Mex., from November, 1888, to April, 1889. Mr. F. H. Newell, later director of the Reclamation Service, and now professor of civil engineering at the University of Illinois, was placed in charge of the camp and presumably directed and assisted the fourteen young men who were selected to take the practical course in what may be called the "First Principles of Stream Gaging." Prof. George E. Curtis of Washburn College, Topeka, formerly in the United States Signal Service, was also an instructor at the camp, and apparently had charge of the meteorological division of the work.

Capt. C. E. Dutton of the Ordnance Corps, U. S. Army, temporarily on duty with the Geological Survey, was the officer in general charge of the organization of the hydraulic survey and the engineering survey, which, with the topographic survey, were undertaken to secure the base data required for the successful prosecution of the immense irrigation projects about to be undertaken in the West.

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* By C. H. Pierce.

The report of Captain Dutton, dated November 1, 1889, contains much of interest to present-day engineers. It should be kept in mind that, in his discussion of methods and equipment, Captain Dutton was referring to the measurement of water in natural stream beds for the purpose of determining the probable average daily flow of those streams, and that he was not concerned with the measurement of water in power canals, or with precise methods of determining the efficiency of water wheels.

In his report Captain Dutton said:

> "The measurement of water flowing in a stream is not an easy matter. It requires skill and rather costly instruments. While the general method has been the subject of much inquiry by a few men, and has been practiced to a small extent, there is a wide variation in details and considerable uncertainty or discordance in results. At the beginning of the organization there were no men available who possessed the requisite experience and skill except two or three men who were occupying responsible positions, and it was doubted if they could be induced to relinquish them. Neither were any instruments to be found in the market as articles in regular supply, and such as were needed must be made to special order. . . . In view of the novelty of the work thus involved upon the Survey, of the impossibility of finding men skilled in the work required, of want of instruments adapted to the work, and in further view of the fact that the winter was near at hand, during which the field work would, in most portions of the West, be impracticable, it was deemed best to select a small body of young men of good education and high general intelligence and establish them at some advantageous station where they could, in the course of the winter months, acquire a knowledge of the methods and instruments they would have to employ. Fourteen young men were carefully selected and were placed in a camp of instruction, situated at Embudo, on the Rio Grande River, about fifty miles north of Santa Fé, in New Mexico, where they passed the winter in practicing with the various instruments selected for trial and in becoming familiar with the theory and practical application of the methods. In the month of April the camp was broken up and the men distributed to their respective fields of work.
>
> "The camp of instruction at Embudo was placed in charge of Mr. F. H. Newell, and the work required of the men consisted in practical stream gaging by various methods, measuring the rise and fall of the stream from day to day, measuring the daily evaporation and making observations with meteorological instruments.

"*Stream Gaging.*

"In measuring the flow of streams, it is to be remembered that the flow varies from time to time. The measurement made on any given day, while the river keeps a steady flow for an hour or two, is good only for that day and hour, and for times when the river is at the same stage. A small rise in the stream is accompanied by a large increase in the flow, and a small fall by a large decrease in the flow. To ascertain the flow during a considerable period of time and thus obtain an average for varying stages, two distinct classes of measurement are necessary: First, the amount of flow corresponding to each and every stage; second, a continuous record of the rise and fall.

"(1) Measurement of flow. — There are four methods of gaging the flow of a stream: (*a*) by weirs; (*b*) by floats; (*c*) by formula; (*d*) by meters.

"Weirs are practicable and economical only in the case of small streams at low water, and in such cases the system is preferable to all others. When the depth of the water passing over the weir much exceeds a foot it becomes sometimes uncertain. In streams not exceeding 25 to 30 ft. in width, and carrying not more than 40 or 50 cu. ft. of water per second, a weir may be constructed quickly and cheaply. It is almost certain to be swept away by high water, but if small and in shallow water it can be easily replaced. It is desirable, however, to select stations where the water above the weir has no sensible 'velocity of approach' other than that produced by the constant movement of water over the weir, as it is difficult to make proper allowance for it.

"Gaging by floats is the crudest and most unreliable of all methods. It consists in throwing floating objects into the water and noting the length of time occupied by the float in moving through a measured distance down the stream. The distance divided by the time is the mean velocity of the motion of the float. But this velocity of the float is not the mean velocity of the stream, which varies in different parts of the width and at different depths. As a rough-and-ready rule the mean velocity of the stream is taken to be about four fifths the velocity of the float. This is no doubt a fair approximation where the cross section of the stream is symmetrical, the current smooth, regular and free from eddies, and its course free from obstructions. But in small or medium-sized rivers such conditions are rare, and the whole method of measurement is liable to great uncertainties, and it is impracticable in large rivers.

"Gaging by formula is based upon the assumption that with a given cross section the mean velocity of flow bears a certain ratio to the declivity down which the water moves. The declivity may be accurately ascertained by the spirit level. With a given declivity the flow will vary with the cross section, being greater when the latter is narrow and deep than when it is broad and shallow. The effect of the form of the cross section has been experimentally investigated by several hydrographers and an empirical formula has been devised for it which seems to give very fair results when the cross section is not of very irregular form and when the bottom is not incumbered with large stones and other obstacles. In general, however, it is of very limited utility.

"Gaging by meters is the most satisfactory and complete method of all. The ordinary current meter is a small wheel which is caused by the current to revolve when immersed. It may be of propeller, windmill, or anemometer form. The axis of the wheel is attached to some device for recording automatically the number of revolutions in a unit of time which corresponds to any given velocity; in other words, every meter must be 'rated.' In all good meters the number of revolutions bears a simple ratio to the velocity of the stream unless the velocity is very small. At very low velocities even the best form of meters becomes irregular and uncertain. They are also inferior to weirs for very small or very shallow streams. In gaging with a meter, the cross section of the stream is subdivided into portions 5 ft. wide in ordinary streams and 10 ft. wide in very broad ones. The form of the bottom and areas of subdivisional cross sections are ascertained by soundings, and the mean velocity of each is measured with the meter. Each subdivision is therefore treated as a separate stream and the flow of the entire stream is the sum of the flows of its constituent parts.

"The object of this camp was to train the new members of the hydrographic division for active and independent field work for the succeeding seasons. The principle under which the operations at the camp were conducted was to attempt all those investigations which seemed pertinent to the future work of the Survey, modifying from time to time the methods and instruments as experience developed obstacles or objections.

"A regular routine of observations was begun and carried on, by which each man in turn became familiar with the details of each investigation; but beyond this the fullest possible scope was left for individual development, and opportunity for original work in such lines of research as each should choose.

"A small rope ferry was established near the camp across a suitable part of the river, and various forms of meter used for obtaining the velocity. Changes and improvements in the meters were suggested while experience was being acquired.

"From the results obtained by the meters the velocities in various parts of the river's cross section were determined and both horizontal and vertical curves of velocity were plotted, making a comparative study under various conditions.

"While meters were being used at one part of the river other methods of stream measurement were tried at suitable points, i. e., by surface floats, subsurface floats and vertical rods. Comparisons were made between the results obtained in these various ways as well as by the formula for river flow given by Kutter.

"Records of the river height were kept continuously and a study made of the relation between river height and discharge. These records were kept both by observation and by an automatic recorder or 'nilometer' which was installed and tested as to its suitability for this work."

Beginning with the experimental work at the Embudo camp, and during the subsequent work which was soon being carried on in different parts of the country, it was seen from the vertical curves of velocity that certain general laws applied to the velocity of the moving water in a vertical section of a natural stream. At the present time these laws are better understood, and are generally covered by a statement that, in a section of a river where conditions are suitable for current-meter work, the variation in the magnitude of the velocities in a vertical section may be closely represented by a parabola, the axis of which is, in general, below and parallel to the water surface. The exact size and shape of this parabola of velocities will continually vary throughout the width of the stream, so it follows that the measuring sections must be taken sufficiently close together to determine the variation of velocity in the horizontal section.

From the general shape of the vertical curves of velocity it was thought that the average velocity in the vertical section could be determined from a measurement of the velocity at some one point in the section, and a large amount of experimental work was done to determine the coefficient to be applied to reduce the observed velocity to a mean for the vertical section.

It must be remembered that the scope of the work was such that whatever methods were used must be applicable to streams varying greatly in size, some gagings being made where the depth of water was not more than 1 foot, whereas other gagings must be made where the depth was 40 or 50 feet, the hydrographer in some cases working from a bridge perhaps 25 or 30 feet above the water surface, and in other cases working from a car swaying from a cable in mid-air. He usually works alone, and must make accurate soundings of the depth from water surface to river bottom; he must place and hold the meter at the proper point or points for the observations; he must count the number of revolutions of the meter and obtain the corresponding time by the stop-watch; he must keep his note-book convenient and record the results at the end of every observation; and, in the case of high velocities, he may find it necessary to operate a stayline to keep the meter in the proper position in the vertical section.

Under these conditions of the problem, it was necessary to simplify methods as much as possible, and although it was realized that observations of velocity at many points in the vertical section might be desirable, that method was generally impracticable on account of the time required. Any advantage due to ultra-refinement in the determination of velocity in any one section might be entirely lost by a change in stage of the river during the extra time required by the ultra-refinement.

It was early seen that for a single observation taken at 0.6 of the depth the coefficient to reduce to mean velocity was generally very close to unity, the difference being positive about as often as negative, and generally not exceeding a few hundredths. This 0.6 depth method was accepted as the standard for several years, although frequent vertical curves were obtained and numerous studies and comparisons were made.

During 1901 and 1902 a series of observations were made on the flow of rivers in the vicinity of New York City, the results of which are described in U. S. G. S. Water-Supply Paper No. 76, which is illustrated by numerous curves and diagrams showing the shapes of velocity curves for various types of streams and different conditions of stream beds.

In 1903 the writer made numerous comparisons and plotted many vertical curves of velocity for streams in Vermont, and engineers in other sections of the country were making similar tests of the method. In 1904 the second and enlarged edition of "Accuracy of Stream Measurements," by E. C. Murphy, was published as U. S. G. S. Water-Supply Paper No. 95. Besides giving the results of Murphy's Cornell experiments, this paper contains tabulations showing the results of a large number of measurements by different methods and under various conditions. Soon afterward, in 1905, the method known as the "0.2 and 0.8 depth method" was brought out, as being more reliable than the single-point method, and yet simple enough to be easily used. In addition to a theoretical proof, thousands of tests and comparisons of this method have been made, a summary of which is contained in Hoyt and Grover's textbook on "River Discharge." This textbook should be referred to by any one desiring detailed information as to the results of the comparisons; the general summary shows that of 476 curves the average coefficient needed to reduce the mean of the 0.2 and 0.8 depth observations to the true mean was 1.001, the highest coefficient for any curve being 1.026 and the lowest coefficient 0.970. This shows an average error of one tenth of one per cent. with a maximum individual error of 3 per cent.

Mr. R. A. Hale, of Lawrence, Mass., has contributed valuable data showing the results of the use of the 0.2 and 0.8 depth method in rectangular flumes and canals. Twenty-three measurements made in five different conduits show that the average coefficient needed to reduce the mean of the 0.2 and 0.8 depth observations to the true mean was 0.993, the highest coefficient for any curve being 1.028 and the lowest coefficient 0.945.

The use of the current-meter should presuppose a knowledge of its proper use. Like most scientific instruments, it can only be used to advantage by those familiar with the instrument and with its limitations. While it cannot be expected to compete with the chemical or electrochemical methods of measurement of water through turbines, on the other hand those latter methods cannot compete with the current meter in obtaining the data on the run-off of rivers which are generally needed for water-power studies. With a method of measurement which gives results averaging less than one per cent. in error it would seem that further refinement of methods would be unwarranted, considering the uncertainties which enter into any water-power study, and the fact that previous conditions of flow are not likely to be exactly duplicated in the future.

An interesting and instructive discussion of the history and vicissitudes of the current meter in the United States is contained in the Proceedings of the Engineer's Society of Western Philadelphia for June, 1914, where Mr. N. C. Grover describes current meter work and also gives an interesting account of the work of the hydrographer and a description of the varied conditions he must be prepared to meet.

In conclusion it may be said that the 0.2 and 0.8 depth method which is now widely used is the result of many years of study and investigation by engineers in all parts of the United States. Since, however, this method has been adopted as an improvement over other good methods, it is to be hoped that still further improvement may be made, and that any errors still existing may be finally eliminated.

0.2 AND 0.8 METHOD OF CURRENT METER MEASUREMENT IN POWER CANALS.*

A long, straight section of river or power canal, where the alignment is good and where the sides and bottom are free from obstructions and fairly smooth, thus giving the water a good chance to flow uniformly and without eddies and whirls, offers the best chance for accuracy with the 0.2 and 0.8 method of current-meter measurement.

Conditions approaching the ideal are found most readily in the larger rivers with sandy bottoms — never in mountain streams and very rarely in power canals.

In any case, after a section has been selected with due care for manner of approach of the water, freedom from backwater and as good conditions with ice cover as possible, multi-point traverses should be made in order to establish definitely whether the 0.2 and 0.8 method may be used in future measurements, or whether the 0.6 or integrating methods will give better and more accurate results. Where the section and alignment is good, the bottom and sides smooth, and water is not fed in or taken out irregularly, the 0.2 and 0.8 method may be found to give accurate results.

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* By Arthur T. Safford.

In the case of power canals, such conditions are rarely encountered. Power canals are generally relatively short, water is fed in at the head, sides or both, through gates, one or more of which may be wholly or partly open at different times, and water is taken out at the end or at various places along the canal in all manner of ways. In addition, such canals are liable to be bridged at short intervals with the bridges carried on piers or posts, to contain ice or log booms, and even to be crossed by pipe lines at or near the bottom. All these and similar obstructions cause eddy currents and whirls, and even the smallest obstruction may cause a large variation in the normal distribution of velocities. The 0.2 and 0.8 method may in such cases be found to be several per cent. high or low, even if the vertical sections are taken very close together.

There are many examples of velocity distribution in power canals which might be cited to prove the inaccuracy of using the 0.2 and 0.8 method without having previously made multi-point measurements to determine accurately the velocity distribution. A few examples of such conditions follow. (Table 1.)

During the test of a 54-in. R. H. McCormick wheel at the Holyoke Water Power Company testing flume, October 16, 1900, the results obtained for head and discharge were very carefully investigated. During the actual test of the wheel at full gate opening, about 14.6-ft. head and from 108 to 109 R.P.M., the load was held constant long enough for measurements to be made from a horizontal straight-edge down to the water-surface to check the agreement of the two hook gages, and at the same time two multi-point current meter measurements were made, one 6 ft. up from the weir, the other 10 ft. up from the weir or directly over the 1-in. pipe to the still-boxes. The horizontal straight-edge consisted of a beam across the flume above the 1-in. pipe with seven nails set 1, 4, 7, 10, 13, 16 and 19 ft. from the left side of the flume and very carefully leveled. Readings were made from these nails to the water-surface by use of a hand-hook gage. Readings were also made at three other points 16 ft.

TABLE I.

Comparison of Mean Velocities with 0.2 and 0.8 Velocities in Power-Canal Measurements.

**Pawtucket Canal — Lowell — at Upper Side of Hamilton Footbridge to Blue Dye House. March 29, 1918,* P.M.

Station.	Depth. Feet.	Mean Velocity. Ft. per Sec.	Velocity. Ft. per Sec.		Mean Ft. per Sec.	Ratio Mean of 0.2 and 0.8 Velocities to Mean Velocity in Per Cent.
			0.8 Depth.	0.2 Depth.		
3	9.5	3.83	3.09	4.12	3.61	94.2
10	11.7	3.26	2.48	4.04	3.26	100.0
17	11.6	4.87	4.83	4.24	4.54	93.2
27	12.3	5.59	5.91	4.90	5.40	96.6
36	12.4	5.05	4.73	4.97	4.85	96.0
45	12.0	4.71	4.12	5.24	4.68	99.4
54	12.0	4.56	4.13	5.71	4.92	108.0
63	12.0	3.88	3.35	4.29	3.82	98.4
72	11.7	3.50	2.70	4.23	3.47	99.1
81	9.4	3.45	2.64	3.95	3.30	95.6
90	7.1	3.05	2.74	3.28	3.01	98.7
95	5.8	2.99	2.26	3.48	2.87	96.0
		4.06			3.98	98.0
Lawrence Manufacturing Company. Nos. 4, 5, 6 Raceways. July 26, 1910. Right-hand Raceway No. 16–151.						
1	1.70	6.050	6.25	5.85	6.05	100.0
2	1.70	6.696	6.79	6.74	6.765	101.1
3	1.70	6.279	5.91	6.92	6.415	102.2
4	1.70	6.633	6.62	6.93	6.775	102.2
5	1.70	6.530	6.45	6.77	6.61	101.3
6	1.70	6.335	6.14	6.75	6.445	101.9
7	1.70	6.392	6.30	6.77	6.535	102.3
8	1.70	6.169	6.06	6.51	6.285	102.0
9	1.70	5.891	5.70	6.24	5.97	101.5
10	1.70	5.482	5.40	5.74	5.57	101.6
11	1.70	4.818	4.84	4.97	4.905	101.9
		6.116			6.211	101.6
August 5, 1910. Left-hand Raceway No. 18–151.						
1	1.81	3.544	3.45	3.57	3.51	99.0
2	1.81	5.514	4.85	6.32	5.585	101.4
3	1.81	6.300	5.71	6.95	6.33	100.5
4	1.81	5.909	5.27	6.63	5.95	100.7
5	1.81	6.249	5.56	6.87	6.215	99.4
6	1.81	5.558	5.15	5.98	5.565	100.2
7	1.81	4.569	4.42	4.55	4.485	98.1
		5.378			5.377	99.9

* Not exact.

TABLE 1. — *Continued.*

Saco-Lowell Shops. Nos. 1 and 2 Wheels. October 24, 1916, P.M.

Station.	Depth. Feet.	Mean Velocity. Ft. per Sec.	Velocity. Ft. per Sec.		Mean Ft. per Sec.	Ratio Mean of 0.8 and 0.2 Velocities to Mean Velocity in Per Cent.
			0.8 Depth.	0.2 Depth.		
1	5.76	2.15	1.99	2.28	2.145	99.8
4	5.76	2.25	2.19	2.31	2.25	100.0
7	5.76	2.19	2.10	2.22	2.16	98.6
10	5.76	1.72	1.59	1.82	1.705	99.1
13	5.76	1.36	1.24	1.48	1.36	100.0
16	4.94	1.15	1.07	1.20	1.135	98.7
18	4.44	0.88	0.90	0.93	0.915	104.0
		1.67			1.667	100.0
Lawrence Manufacturing Company. Nos. 1, 2, 3 Races. August 5, 1910. Left-hand Race.						
0.3	2.60	1.307	1.45	1.22	1.335	102.1
2.	3.80	1.514	1.58	1.50	1.54	101.7
4.5	5.90	1.814	1.88	1.84	1.860	102.5
7.	6.50	1.656	1.66	1.78	1.72	103.9
9.5	5.70	1.749	1.64	1.64	1.64	93.8
12.	3.90	1.630	1.53	1.77	1.65	101.2
13.9	1.80	1.409	1.33	1.55	1.44	102.2
		1.583			1.598	101.1
Right-hand Race.						
0.3	2.20	0.468	0.63	0.36	0.495	105.7
2.	4.00	0.989	0.98	1.02	1.00	101.1
4.5	5.80	1.206	1.20	1.14	1.17	97.0
7.	6.60	1.226	1.19	1.28	1.235	100.7
9.5	5.50	1.339	1.33	1.38	1.355	101.2
12.	3.50	1.319	1.32	1.34	1.33	100.9
13.9	1.80	1.035	1.02	1.09	1.055	102.0
		1.083			1.091	101.2
Bigelow Carpet Company at Entrance to Feeders. October 5, 1911.						
L. H. Feeder........		5.23	4.35	6.67	5.51	105.2
Middle Feeder......		4.45	4.53	4.50	4.51	101.5
R. H. Feeder.......		5.00	5.84	4.50	5.17	103.5
		4.89			5.06	103.4
**Holyoke Testing Flume — 10 Ft. up from Weir. October 16, 1900.*						
1.	8.052	1.27	1.25	1.51	1.38	108.6
4.6	8.052	1.50	1.50	1.80	1.65	110.0
8.2	8.052	1.76	1.84	1.84	1.84	104.5
11.8	8.052	1.57	1.70	1.78	1.74	110.8
15.4	8.052	1.26	1.36	1.22	1.29	102.4
19.	8.052	1.30	1.15	1.55	1.35	103.8
		1.475			1.577	107.0

* Not exact.

TABLE 1. — *Continued.*

**Holyoke Testing Flume — 6 Ft. up from Weir.*
October 16, 1900.

Station.	Depth. Feet.	Mean Velocity. Ft. per Sec.	Velocity. Ft. per Sec.		Mean Ft. per Sec.	Ratio Mean of 0.8 and 0.2 Velocities to Mean Velocity in Per Cent.
			0.8 Depth.	0.2 Depth.		
1.	8.052	1.12	0.64	1.46	1.05	93.7
4.6	8.052	1.43	1.25	1.48	1.37	95.8
8.2	8.052	1.75	1.60	1.90	1.75	100.0
11.8	8.052	1.68	1.45	1.89	1.67	99.4
15.4	8.052	1.34	1.22	1.37	1.29	96.2
19.	8.052	1.32	1.13	1.62	1.38	104.5
		1.484			1.459	98.3

**Pawtucket Canal — Lowell — at Upper Side of Hamilton Footbridge to Blue Dye House. May 23, 1918,* P.M.

3	9.1	3.40	2.81	3.71	3.26	95.9
10	11.3	3.77	3.09	4.85	3.97	105.3
17	11.2	5.06	5.00	4.78	4.89	96.6
27	11.9	5.29	6.00	4.48	5.24	99.0
36	12.0	5.79	5.00	6.31	5.66	97.7
45	11.6	5.21	4.60	5.77	5.18	99.4
54	11.6	5.23	4.90	5.52	5.21	99.6
63	11.6	4.56	4.00	4.96	4.48	98.2
72	11.3	4.00	3.30	4.70	4.00	100.0
81	9.0	3.86	3.30	4.52	3.91	101.3
90	6.7	3.48	3.22	3.56	3.39	97.4
95	5.4	2.97	2.87	3.06	2.97	100.0
		4.49			4.46	99.3

NOTE. — It is evident from an inspection of these results that while the actual velocity is usually but within a few per cent. of that obtained from the 0.2 and 0.8 method, there is no assurance that the latter method can be relied upon, and it is therefore better for each section to make complete traverses of velocities and a comparison in advance, leaving the 0.2 and 0.8 method for complete measurements at a later date. The committee will endeavor to have in the final report some comparisons between the two current meter methods and some rod float measurements in rectangular canals, which have been made in Lowell for a great many years.

* Not exact.

from the left side of the flume, one 6 ft., one 19½ ft. and one 30 ft. up from the weir. The mean of 130 readings showed the right-hand hook gage 0.0075 ft. lower than the left-hand hook gage, and the average depth on the weir to be 2.2000 ft. At the same time six sets of measurements from nails to water surface gave 2.2173 ft. as the average depth on the weir, or 0.0173 ft. more than shown by the hook gage. A comparison was also made between the readings of the left-hand hook gage and the surface of the water at the different points from 6 to 30 ft. up from the weir. The water surface in every case was lower 6 ft. up from the weir than it was 10 ft. up, but both these points stood higher than the hook gage showed. Multi-point current meter measurements were also made at sections 6 ft. and 10 ft. up from the weir. The average velocity 10 ft. up from the weir, or at the hook-gage pipe, was found to be about 1.47 ft. per second; and with the holes (in the bottom and 3 ft. apart) at right angles to the current, the head is apparently held down about three-quarters as much as the head (0.03 ft.) required to get up this velocity. Had this point measurement been made simply at 0.2 and 0.8 depth, the resulting average velocity in the section would have appeared to be about 1.57 ft. per second, or nearly 7 per cent. too high. For the measurement 6 ft. up from the weir, the 0.2 and 0.8 method shows about 98.3 per cent. of the actual average velocity.

A very recent measurement showing the effect of an ice boom has been made in the Pawtucket Canal in Lowell at the upper side of the Hamilton footbridge. Four measurements have been made, on March 15, 22 and 29, 1918, with the boom in place, and on May 23, 1918, with the boom partly removed.

These measurements were multi-point measurements. Had the measurement of March 29 been made only at 0.2 and 0.8 depth, the resulting velocity would have been about two per cent. less than the actual average velocity in the section. Had the measurement of May 23 been made only at 0.2 and 0.8 depth, the resulting velocity would have been about one per cent. less than the actual velocity in the section.

On July 26 and August 5, 1910, some measurements made in the Lawrence Manufacturing Company (Lowell) Nos. 4, 5 and 6 raceways show the effect of a shallow power canal with a rough bottom and fast velocities. There is a very decided pulling back of the water at the *bottom* of the section.

If these measurements had been taken simply at 0.2 and 0.8 depth, the apparent average velocity would have seemed to be about 1.5 per cent. higher than the actual average velocity for the right-hand raceway and about right for the left-hand raceway.

Measurements were also made on the Lawrence Nos. 1, 2, 3 raceways on August 5, 1910. These raceways are much deeper at the center and about the same depth on the edges as the Nos. 4, 5, 6 raceways. Here the pulling-back effect in the center is not as noticeable, due to smoother bottoms and greater depth, but the effect is greater in the shallower parts. The 0.2 and 0.8 method gives results over two per cent. high for the outer sections, while it gives about the correct velocities for the center or deeper part of the canal.

The effect of pulling water out of a canal is shown in the measurement made October 5, 1911, at the entrance to the three feeders of the Bigelow Carpet Company, now the U. S. Cartridge Company, Lowell, Mass., plant. At the entrance to the left-hand feeder the 0.2 and 0.8 method showed over five per cent. higher than the actual average velocity, to the center feeder 1.5 per cent. and to the right-hand feeder 3.5 per cent.

PRECIPITATION, EVAPORATION AND RUN-OFF.*

Rain or snow falling on the drainage areas of rivers is the immediate source of run-off. The relation between the precipitation and the run-off is a direct one; rain falls on the ground and a part of it finds it ways to the water courses, and yet there are so many physical factors affecting the disposal of precipitation that to establish any exact relation between it and run-off is impossible. The cycle of precipitation and run-off is complete when we include the evaporation of moisture in the form of vapor from water and ground surfaces, to fall later as rain.

The U. S. Weather Bureau and private individuals have kept precipitation records at stations geographically widely distributed for a long period of time; and it frequently happens that good precipitation records are at hand when no stream gagings can be obtained. For this reason, any relation, even only approximately fixed, between precipitation and run-off is of considerable value to the engineer. Run-offs based upon a percentage of precipitation are only true of long-term averages and are to be used with the greatest caution. A dry year means one of low precipitation where the amount available for run-off is but a few inches; a wet year one when, with all natural requirements satisfied, there is a large balance for run-off.

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* By Arthur T. Safford.

The local variation of the average annual precipitation in the United States is all the way from nothing in some years in the desert regions to an occasional maximum of more than 100 ins. in the mountains of the extreme northwest. In general, mountain ranges receive a greater amount of precipitation than the surrounding valleys and yield a greater proportionate run-off. The lack of mountain records has kept this fact from being common knowledge.

The main factors affecting the disposal of precipitation are the topography, geology, evaporation and the amount taken up by vegetation. Rugged drainage areas with steep slopes discharge the water readily and quickly into the water courses. Rivers draining such areas, unless there is good storage in lakes and ponds, are more subject to sudden fluctuations in flow than rivers draining flat or rolling country where there are many swamps and where the water has chance to sink into the pervious soil. Evaporation, according to our best present knowledge, is essentially the same from year to year in any one locality but varies more or less regularly with the temperature of the water and wind; the amount of moisture taken by the vegetation is about a constant.

Precipitation records have been kept in a few places for long periods of time and yet they have not been collected and analyzed in any exhaustive way until recently. X. H. Goodnough, in a paper published by the New England Water Works Association in September, 1915, has collected rainfall statistics from all over New England from the earliest known, 1749, to the date of writing his paper. Detailed consideration cannot be given here to the variation of precipitation over long periods of time; but mention must be made of the geographical distribution of rainfall in New England, and in other districts where adequate records are available.

*"The highest rainfall recorded at any place in New England is at the summit of Mt. Washington, where observations were made continuously from September, 1871 to 1887. The average rainfall during that period was 83.53 in. The lowest rainfall recorded in New England is that along the shores of Lake Champlain, where at Burlington, Vt., the average rainfall for a very long period of years has amounted to 32.76 in.

"It is evident from a consideration of the available records that the effect of elevation upon the rainfall is very marked, and it is probable that the higher rainfall in the mountainous districts has not hitherto always been adequately considered in studies of the flow of streams and yield of watersheds. Unfortunately, except at the summit of Mt. Washington, few rainfall stations have been maintained among the higher ranges either in the White Mountain region or the other mountainous regions of New England, so that it is impossible to ascertain definitely the extent of these areas of high rainfall. In the White Mountain region the very high rainfalls are very probably confined to the highest summits and diminish rapidly as lower levels are reached, though the records available show very clearly that the rainfall south and east of these mountains is higher than elsewhere in this region except on Mt. Washington. There are indications also that the rainfall in western Maine along the easterly slope of the extensive highlands west of the Kennebec River is higher than in the central and northern portions of the state.

"The rainfall at Bar Harbor at the eastern base of the mountains of Mt. Desert is much higher than elsewhere in this region, and it is possible that along the highlands toward the north of Bar Harbor and east of Bangor similar conditions obtain, but the available records are inadequate to show, except in a very general way, the variations in rainfall in that region.

"Aside from the higher rainfall of the mountainous districts, the highest precipitation in any large area in New England occurs in the southern portion of the district. This area of high rainfall extends from the neighborhood of the Croton watershed in New York, where the rainfall is higher than in adjacent portions of New England, across Connecticut, Rhode Island and

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* Copied from X. H. Goodnough's paper on "Rainfall in New England, *Journal of the New England Water Works Association*, September, 1915, page 248.

parts of southeastern Massachusetts, nearly to the eastern coast of the latter state, the amount gradually diminishing as a rule from approximately 50 in. in western Connecticut to about 46 in. in southeastern Massachusetts."

This excess of precipitation due to high altitudes should not be lost sight of; the gathering ground of many of our best reservoir sites is made up of high rocky and wooded areas both well situated for a large run-off; if reservoirs are deep and of large capacity, the net result may be a very large part of the precipitation actually collected.

In the Catskill Mountains gages at the summits invariably show materially greater precipitations than in the valleys, the difference being as much as 15 ins.

The seasonable distribution of precipitation has marked effect on the proportionate part reaching the streams. Studies of precipitation and yield in different areas, where accurate records of precipitation and run-off have been kept, demonstrate that for the precipitation to reach the streams abundantly it must fall in the months from October to April inclusive and not from May to September, when growing crops take the greater part. A well-sustained summer flow is only possible where an excess of precipitation prevails.

The Mississippi valley, extending from the Gulf of Mexico to Canada, and from the Alleghanies to the Rocky Mountains, is shut in on the east by mountain ranges whose eastern and southern slopes receive most of the rain borne by easterly winds blowing off the Atlantic, and, robbed by the western slopes of the Rockies of most of the rain borne by westerly winds from the Pacific, the source of most of the rain falling over the Mississippi valley is to be sought in the Gulf of Mexico. Within this basin the southern extremity receives the heaviest precipitation while the extreme northwesterly portion, at the headwaters of the Missouri, receives but little more than one fifth the amount falling at the mouth of the river.

This is indicated by the following table, which shows that, independent of altitudes, there is a tendency for the greatest rainfall to fall near the coast, and gradually to decrease as the wind currents travel inland.

	Average Annual Precipitation. Ins.
New Orleans	60
Memphis	52
Cincinnati	42
Pittsburgh	36
St. Louis	40
Headwaters upper Mississippi	25
Headwaters upper Missouri	13

Precipitation is the only source of run-off; evaporation is, after vegetation, the chief agency reducing the available supply of water running off in the rain-water courses. Wherever there is storage of water in natural ponds or artificial reservoirs, the effect of evaporation from water surface must be considered; and in some cases, which the irrigation projects typify, evaporation from the soil must be considered as well.

TABLE 2.

EVAPORATION IN INCHES FROM WATER SURFACE AT CERTAIN LOCALITIES IN MASSACHUSETTS, MAINE, CALIFORNIA, NEVADA AND OKLAHOMA, BY MONTHS.

	Chestnut Hill, Boston, Mass. Inches.	Maine Composite Figures. Inches.	Lake Tahoe, California, 1901. Inches.	Sweetwater Reservoir, California. Mean 1910–14. Inches.	Reno, Nevada, 1901. (Seven months only.) Inches.	Lake Lawtenka, Oklahoma, Mean 1913–15. Inches.
January	0.96	0.7	0.84	2.56	....	1.665
February	1.05	0.7	0.70	2.56	....	1.825
March	1.70	1.1	0.77	2.73	....	3.42
April	2.97	1.6	1.25	4.08	....	4.38
May	4.46	2.1	2.42	6.09	2.80	6.11
June	5.54	2.8	3.35	6.74	5.35	7.90
July	5.98	4.32	4.00	8.80	8.45	10.04
August	5.50	5.0	6.50	7.45	9.12	9.24
September	4.12	3.32	4.12	6.62	7.44	6.845
October	3.16	2.2	2.65	5.06	4.31	4.25
November	2.35	1.3	2.09	3.11	2.75	1.98
December	1.51	0.7	1.44	2.18	....	1.23
The year	39.20	25.84	30.13	56.98	40.22*	58.895

* Evaporation at Reno, Nev., from May to November inclusive, slightly exceeds the total evaporation for the year at Chestnut Hill, Boston, Mass.

Records taken in various parts of the country make it possible to take account intelligently of the amount of loss due to evaporation in the neighborhood of the observation station; but so many variable factors of climate and elevation affect the result that records in one part of the country cannot safely be applied to a remote section.

The best-known experiments are those of Desmond FitzGerald at the Chestnut Hill Reservoir, in Boston, Mass., but other experiments and observers in other parts of the country have in recent years been increasing the fund of information.

The best records at this time known to the writer are given in Table 2.

In connection with the Sweetwater experiments, observations indicate that for a given mean monthly temperature the amount of evaporation increases with the elevation above sea level.

Table 3 gives the total annual evaporation for 17 stations in Texas, New Mexico and Oklahoma.

Experiments at Chestnut Hill and other stations indicate that each 9 degrees Fahr. of change in water temperature corresponds to about 1 in. per month change in the amount of evaporation.

For a water-storage proposition, the best conditions for a large net yield are:

1. A high collecting ground near one of the oceans or the Great Lakes.
2. A drainage area large enough to fill the reservoir, almost every year.
3. A deep reservoir and one of small evaporation surface.
4. A high altitude.

(1) and (2) will control the inflow; (3) and (4) the outflow.

The large lake or pond with small drainage area which has no outlet is usually simply a body of water which has no compensating yield to offset the evaporation.

Distribution of run-off: For comparative purposes it is convenient and advantageous to express the flow of rivers not by total flow, but in second-feet per square mile of drainage area.

TABLE 3.

EVAPORATION, TEMPERATURE AND ELEVATION.
TEXAS, NEW MEXICO AND OKLAHOMA.

Place.	Date.	Elevation. Feet.	Evaporation. Inches.	Temperature.
Galveston, Tex.	July, 1887 — June, 1888	6	46.0	68.7
Corpus Christi, Tex.	July, 1887 — June, 1888	20	38.8	69.4
Brownsville, Tex.	July, 1887 — June, 1888	50	37.0	71.0
Rio Grande City, Tex.	July, 1887 — June, 1888	230	53.1	71.4
Palestine, Tex.	July, 1887 — June, 1888	500	47.1	64.1
Austin, Tex.	Jan. — Dec., 1911	600	50.92	69.2
San Antonio, Tex.	July, 1887 — June, 1888	700	52.4	66.8
Lake Lawtonka, Okla.	April, 1913 — Mar., 1914	1,200	60.38	61.2
Lake Lawtonka, Okla.	April, 1914 — Mar., 1915	1,200	55.78	59.4
Abilene, Tex.	July, 1887 — June, 1888	1,740	54.4	62.1
Carlsbad, N. Mex.	Jan. — Dec., 1899	3,000	54.37	63.1
Carlsbad, N. Mex.	Jan. — Dec., 1901	3,000	43.26	64.1
Lake Avalon, N. Mex.	July, 1909 — June, 1910	3,200	94.51	63.8
El Paso, Tex.	Sept., 1889 — Aug., 1890	3,700	81.5	64.8
El Paso, Tex.	June, 1889 — May, 1890	3,700	84.3	65.7
El Paso, Tex.	July, 1887 — June, 1888	3,762	82.0	63.6
Elephant Butte, N. Mex.	July, 1909 — June, 1910	4,250	86.95	61.1
Port Davis, Tex.	July, 1887 — June, 1888	4,930	91.4	59.7
Albuquerque, N. Mex.	Feb., 1900 — Jan., 1901	5,000	77.87	56.5
Albuquerque, N. Mex.	Jan. — Dec., 1903	5,000	87.90	54.1
Fort Stanton, N. Mex.	July, 1887 — June, 1888	6,150	77.0	50.0
Santa Fé, N. Mex.	May, 1913 — April, 1914	6,900	58.92	48.9
Santa Fé, N. Mex.	July, 1887 — June, 1888	7,013	79.8	48.9

The following is a list of the major factors influencing the distribution of run-offs.

(*a*) Geology — is the soil pervious or impervious?

(*b*) Topography — is the drainage area flat or rolling, or has it steep, precipitous slopes?

(*c*) Size — of the contributing area.

(*d*) Climate — amount and distribution of rainfall and evaporation.

(*e*) Storage — whether natural or developed, the degree of development and the location of the storage basins.

(*f*) Forest cover — extent and character.

These cannot be said to be of equal importance, and the relative effect will differ on different areas. The fifth (*e*) and sixth (*f*) factors only can be varied by man; and, looked at in this light, they assume an importance of greater moment than the rest. An even distribution of run-off is a most desirable characteristic.

Taking up in order the main factors, we shall see how each one affects the distribution.

Pervious soil will allow the water falling on it to percolate down into it, where it is held as ground water, a part to find its way into the streams when the general water table is lowered. If the soil is impervious, water must run off quickly and find its way into the water courses. Thus heavy storms of short duration falling on frozen or saturated ground will cause sudden rises in the streams, and much water may be wasted. On our large rivers the run-off in a few weeks may be greater than that of the entire dry period of six months.

Flat or rolling country combined with a pervious soil is admirably fitted to hold the rain in storage as ground water. The precipitous rocky slopes are equally fitted to discharge the rain almost as it falls and do nothing to even up the distribution of run-off. But if complete or nearly complete water storage is provided, a precipitous drainage area may be better for water-storage purposes than more level country. Between the two conditions are a multiplicity of possible combinations and effects.

The size of drainage area has an additional, sometimes a controlling, influence when there is little or no storage available. This is because the variation in the extent and intensity of rain storms is extreme. A local storm of sufficient intensity to cause a freshet on a small area, occurring on a drainage area of a thousand square miles or more, would hardly be felt on the main river but it may help to keep up the flow of the larger area. It is for this reason that, without storage, small rivers show much greater divergence between maximum and minimum discharge than large rivers.

This would to some extent be offset by a complete forest cover, which on the small stream would undoubtedly delay the peak of the flood and might entirely prevent a sudden rise, and certainly detains the last of the snow and ice for at least a month. It can never, however, hold the water in storage like a reservoir, to be let down at certain seasons of the year. Furthermore, on drainage areas of considerable size, sufficient areas usually cannot be deforested or grow up fast enough to have a more than local effect.

The climate prevailing on a drainage area will depend on its geographical location and on its topography. It is sufficient to refer here to but two aspects, — (*a*) the total annual precipitation and its distribution; (*b*) the total evaporation and its distribution. There is such radical variation in different parts of the country that every case must receive careful study to ascertain the local conditions and tendencies.

Typical hydrographs showing the monthly average stream flow in second-feet per square mile of drainage area naturally exhibit different characteristics according to the local climatic and other conditions. Furthermore, the hydrographs of the same river vary materially from year to year, between rather wide limits.

Experience has shown that, without large water storage, water-power streams cannot be trusted to turn out a given product in any one year; and the breaking-in process of a large development without storage may severely tax the patience and ingenuity of the man operating such a plant.

THE EFFECTS OF ICE ON RIVER DISCHARGE.*

Rivers may in general be divided into two classes: (*a*) those having deep channels with surfaces for considerable distances unbroken by falls or rapids and which are similar to canals in their flow characteristics, and may be called the "canal" type; and (*b*) those having irregular channels with frequent changes in slope, together with numerous riffles, rapids and waterfalls, which may be called the " cascade " type. Naturally the same stream may change from one type to the other in different parts of its course, and the two types may merge so that deep, still sections of the river may be immediately followed by rapids and falls.

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* By C. H. Pierce.

With respect to physical requirements for suitable current meter gaging stations, all rivers may be said to fall between the limiting cases of (1) the canal, with no definite control section, the stage-discharge relation depending upon the slope of the canal, the shape of its cross-section, and the roughness of the sides and bottom, according to the generally accepted formulas of $q=av$, and $v=c\sqrt{rs}$; and (2) the weir, with the water flowing over a well-defined crest or control, for which the discharge may be represented by an equation in the form of $q=clh^n$.

Although the natural stream may never exactly fulfill the theoretical requirements whereby the flow can be accurately computed by either of the above methods, yet for the purpose of this discussion they may be considered as limiting cases, which may be sometimes approximated, and it may be ascertained if in those cases it is possible to deduce any general laws applicable to the effect of ice cover.

Surface ice will be considered first, and then anchor ice, frazil ice, and the occurrence of one or both in conjunction with surface water.

In the case of the canal with surface ice cover, with gage heights read to the water surface, the effect of ice is to decrease the effective area or cross-section for a certain gage height and to increase the frictional resistance, since the wetted perimeter p is increased by an amount equal to the surface width.

For a certain discharge Q, the depth of water or gage height is H, the width of stream is w, and the wetted perimeter is p. As soon as the stage-discharge relation is affected by the formation of ice there will be, for the same discharge, Q, a greater depth H_1 and a greater wetted perimeter p_1+w (Fig. 1), and it may be shown mathematically that the following relation exists:

$$\left[\frac{\int H_1}{\int H}\right]^x = \frac{p_1+w}{p}.$$

Whereby it may be seen that, theoretically, when complete ice cover is once formed there is an immediate increase in the gage height from H to H_1, corresponding to the increase in the wetted perimeter from p to p_1+w, but that, after the quantity w is once introduced into the equation, slight increases to the numerator p_1+w require only small increments to the difference between H_1 and H, since the left-hand member of the equation is to be raised to the x power where x is greater than unity. Therefore the resulting locus of the plotting of H_1 is a curve always above the curve of H and also always above a curve parallel to the curve of H but approaching this parallel curve as a limit at an always varying distance from the curve of H. Fig. 2 represents the locus of H_1 as it varies with the stage, but with ice conditions remaining the same. For every different set of ice conditions the locus of H_1 would be a different curve, and at varying distances above the "parallel" curve. There is no one curve which would represent the locus of H_1 under the varying ice conditions occurring during the winter, and therefore there is no one curve which can be properly used as a "winter rating curve" when the stage-discharge relation is affected by ice.

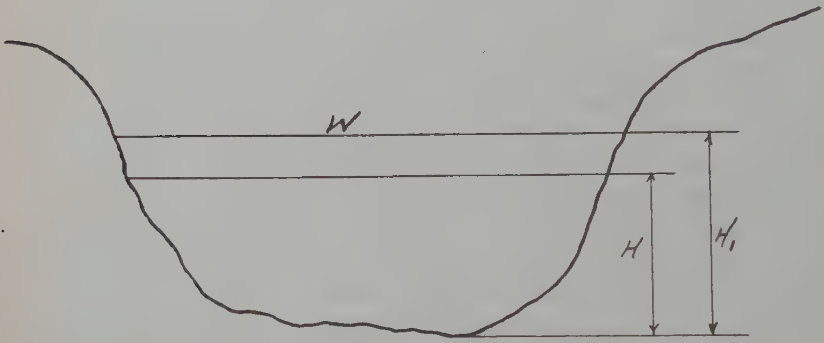

FIG. 1.

In the case of a weir section, or the control section of a stream of the cascade type, the disturbing effect of ice will occur when ice forms on the control irrespective of whether or not the stream may have ice cover, except that if there is ice cover in addition to ice on the control, the backwater effect will be a resultant due to the two causes.

It is difficult to show mathematically how the backwater varies with the stage for stations having a weir control, but it is self-evident that, so long as the ice remains frozen on to the control, the backwater effect will increase up to the point where the ice is completely covered and overflowed, then the locus of H_1 will approach the "parallel curve" as its limit. With ice conditions continually varying, the backwater will also vary, and the only way it can be determined is from the results of discharge measurements.

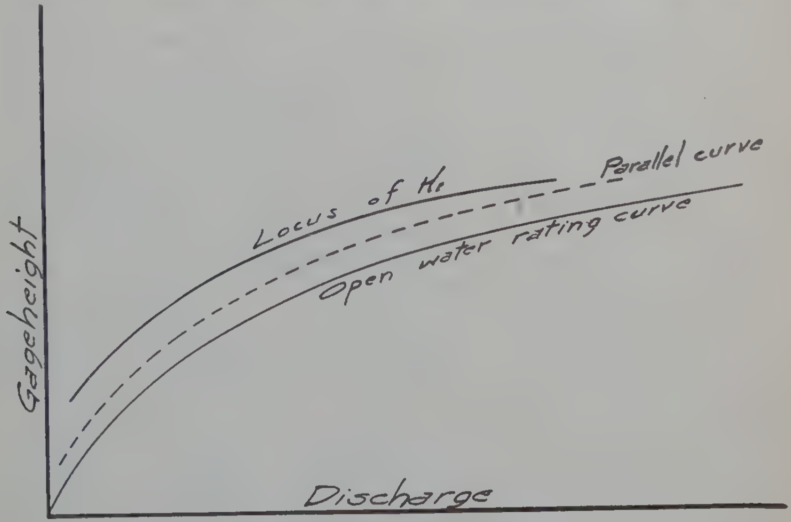

FIG. 2.

Anchor ice and frazil, although occurring from entirely different causes and differing in their methods of formation and appearance, have similar effects on river discharge. The term "slush" ice is commonly applied to both, for the reason that they form a "slushy" mass in the water, the "slush" usually being carried along by the current and frequently collecting in an almost impenetrable mass when the slower velocity has permitted the formation of surface ice.

Although neither anchor ice nor frazil actually form under the cover of surface ice, yet if the temperature is low enough to permit of their formation there is almost certain to be more or less surface ice in places along the stream; only under exceptional conditions of swift, turbulent water, or in canals below gates where a considerable loss of head occurs, would " slush " ice be found without there being also some surface ice. (It should be noted here that both of these conditions are unsuitable for the location of a gaging station.) It is, therefore, generally necessary to deal with " slush " ice in conjunction with surface ice, and it may readily be seen that, with the uncertainties incident to the method of formation and collection of this impediment to the flow, there can be no fixed mathematical relation between the open-water rating curve and the discharge with the channel partially filled with " slush " ice in addition to a more or less complete surface cover. The only mathematical certainty is that the discharge for a certain gage height, with ice in the stream, will always be less than the discharge for the same gage height, with the stream free from ice.

Any method for the successful determination of the river discharge after the formation of ice must be based upon frequent discharge measurement as control points, and some method of filling in between these points. Since the temperature and precipitation, also character of precipitation, whether rain or snow, are essential factors in connection with the formation and change of formation of ice, and also of the amount of run-off to be expected, a study of these data is helpful in connection with the use of discharge measurements.

Method of Determining Daily Discharge at Gaging Stations where the Stage-Discharge Relation is Affected by Ice.

1. Develop an open-water rating curve from measurements at different stages under normal conditions.

This is No. 0 curve in Plate I.

2. From the results of measurements when the stage-discharge relation is affected by ice determine the average curve for these conditions.

This is the No. 10 curve in Plate I.

PLATE I.
(Fig. 3.)
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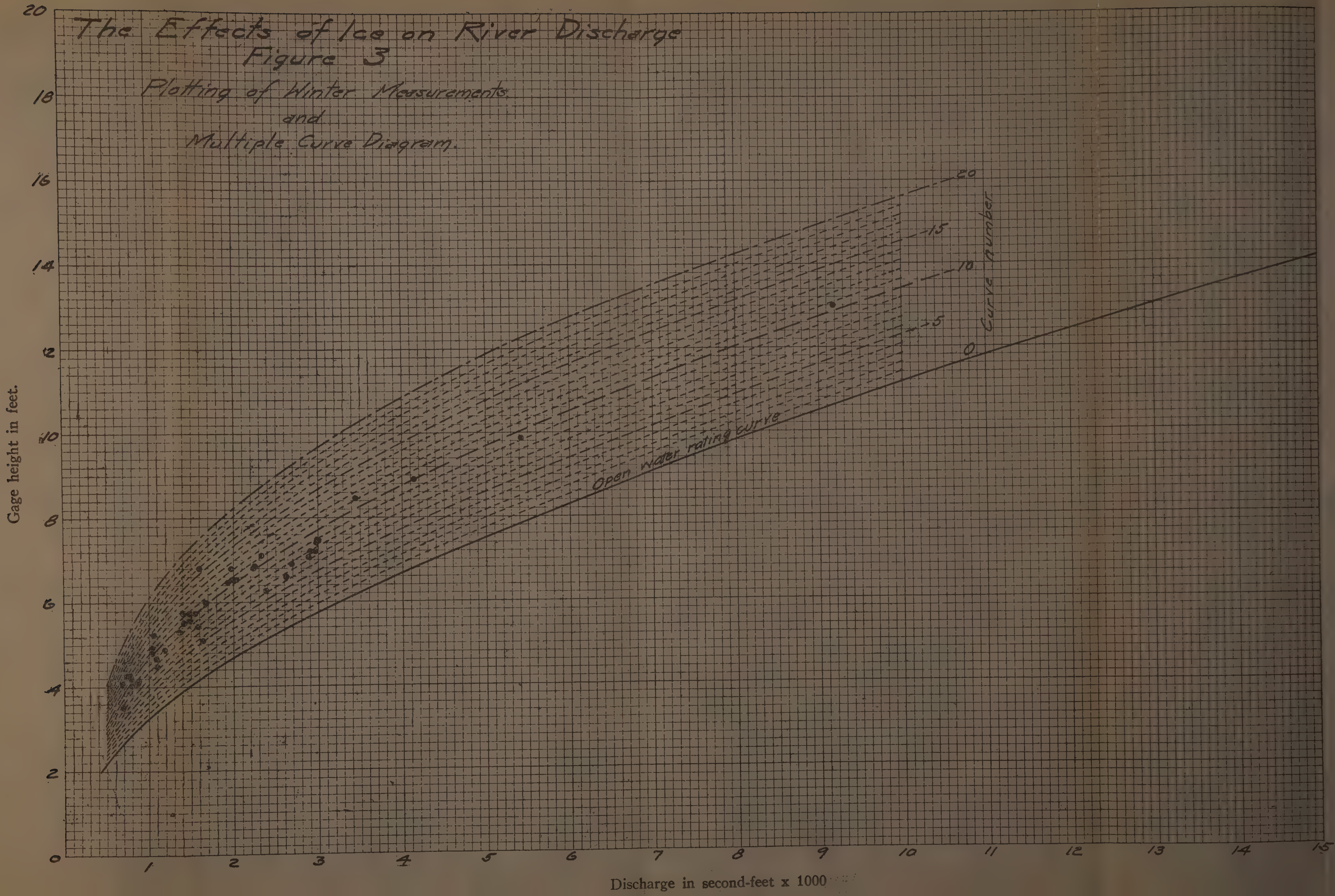

3. Interpolate curves 1 to 9 and exterpolate curves 11 to 20.

4. Ascertain as nearly as possible the date when stage-discharge relation was first affected by ice, and the date when the use of curve No. 0 should cease. This gives point " a " at the beginning of diagram, Plate II.

5. Plot each discharge measurement made during the winter on multiple curve diagram (Plate I), and ascertain thereby the number of the rating curve which is applicable at that time.

6. Plot points corresponding to their numbers, as " b," " c," " d," etc., on diagram, Plate II.

7. Join points a, b, c, d, etc., so that by interpolation the number of the curve to be used on any day may be found.

8. Knowing the number of rating curve and the mean gage height for the day, take out the corresponding daily discharge from the multiple curve diagram or from tables based on the diagram.

The above method is rather tedious, inasmuch as it involves the use of a number of different rating curves or rating tables. If, however, we may assume that the multiple curves are parallel for short portions of their length so that each measurement may be referred to curve No. 0 and a correction to be applied to the mean daily gage height for that day be determined, then by plotting these corrections as ordinates (Plate III) and filling in between the plotted points, either by straight interpolation or, better, by taking into consideration the factors which might influence the corrections, such as temperature, precipitation, description of ice conditions, etc., corrections are obtained which when applied to the observed daily gage heights (Plate III) give the corrected or " effective " daily gage heights (Plate III) to which the rating table may be applied to obtain daily discharge.

This method is the practical application of the theoretical method first described, and is the one which has been generally used by engineers of the U. S. Geological Survey for several years. Numerous studies and comparisons indicate that the results of this method are much more reliable than those obtained by other methods formerly in use.

METHODS TO BE USED IN COMPILATION OF DATA. ANALYSIS OF RUN-OFF RECORDS.*

It will be here assumed that the run-off records will be in the customary form, used by the U. S. Geological Survey, of daily discharge and also of weekly, monthly and annual summaries, the latter giving the maximum and minimum daily discharge, the average for each month, the corresponding second-feet per square mile and run-off depth in inches for each month, and a final summary of the same data for the entire year.

1. Since most hydraulic engineers are now familiar with and extensively use the "daily duration curve," it is suggested that such curves, including the average and "enveloping" curves, be prepared for each gaging station analyzed, covering the entire period of record, the results being presented in both tabular and graphical form.†

If it seems desirable to present all of the curves on one drawing, only the longest term stations should be thus compared, and the duration curves should be for the same period of time.

Attention should be called to the method of preparing duration curves. A duration curve is computed by tabulating the number of days "equal to or greater than" a given rate of flow, and this is exactly opposite to the "deficiency" table or curve which is obtained by tabulating the number of days "less than" a given rate of flow. Thus a duration table (or curve) may be directly obtained from a deficiency table (or curve) by subtracting the number of days corresponding to each rate of flow from the total number of days in the year or entire period.

2. Since a duration curve for a year or longer does not indicate the monthly distribution of flow, and since the latter is a requirement in relay studies, and since furthermore it is just as easy to tabulate the original computations by months and total for the years and period, it is suggested that a secondary summary table (and perhaps curves) be given, indicating the total duration of flow, month by month, for the period of record.

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* By C. H. Pierce.

† For purposes of ready comparison, these curves should be plotted per cent. of time against second-feet per square mile. Arithlog paper is suggested.

3. From the foregoing two summaries, additional tables (or curves) should be prepared, indicating, for each five per cent. of the time, the ratio of the computed to the average rate of flow for the period, either for each year or the total period, or both. For a discussion of this theory see "Power Estimates from Stream Flow and Rainfall Data," by D. M. Wood, in the JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS for March and June, 1916, — especially the latter; also "Comparison of New England Run-off Data," by D. M. Wood, in *S. & W. Public Service Journal* for February, 1917. Comparisons by this method should be for the same period of years.

4. For general comparison of records, it may be helpful to present a diagram showing the total depth in inches of run-off, year by year, for each station analyzed, an example of which method was presented some time ago to the entire committee by Mr. Pierce.

5. Many engineers like to divide the year into the storage, growing and replenishing seasons, and, as a corollary to No. 4, the depth in inches of run-off might be shown by seasons as well as by years. If so, it is suggested that the year be divided December to May, June to August, and September to November, as best applicable to New England.

6. Perhaps no analysis would be complete without some form of river characteristic curve indicating the benefits of storage. If such is to be included in our report, only natural run-off records (i. e., records from which the effect of existing storage has been eliminated) should be used. By computing a mass curve for the driest period of record, a characteristic curve, plotting "required storage" against "minimum regulated flow," can be obtained. If this is done, attention should be called to the fact that it presupposes that the storage will be operated for the purpose of obtaining the greatest primary flow possible, and does not take into account benefits derived from operating the storage each year to the fullest extent possible. In other words, it treats of only one of the two general methods of operating a storage reservoir.

7. As an alternative to No. 6 it may be deemed useful to present a tabular comparison by stations of minimum consecu-

tive periods, such as week, two weeks, three weeks, month, two months, etc., up to six months.

It might be suggested that the simple "old-fashioned" hydrograph, obtained by plotting consecutive values of daily discharge, represents more truly than any other method the actual run-off conditions.

8. It might prove interesting to tabulate for a large number of power plants in New England the installed capacity per square mile of drainage area per foot fall of effective head, i. e., unit capacity installed. This would serve to bring out the large variation in practice. The tabulation might well be grouped by plant types.

9. Some analysis should be given of the studies made of winter conditions, both as regards (1) snow storage and (2) the effects of ice on stream-gaging records. As these problems are being treated by special subcommittees, no suggestions are made by this committee.

10. For the treatment of "pondage," attention is called to two methods covered in current literature, the second really being adapted from and suggested by the first:

(1) Chas. T. Main's method, as given in *Journal of New England Water Works Association*, Vol. 21, p. 214.

(2) D. M. Wood's method, as given in JOURNAL OF BOSTON SOCIETY OF CIVIL ENGINEERS, Vol. 3, p. 99.

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